Interrelationships between Reinforcing Bar Physical Properties and Seismic Demands

by J. McDermott

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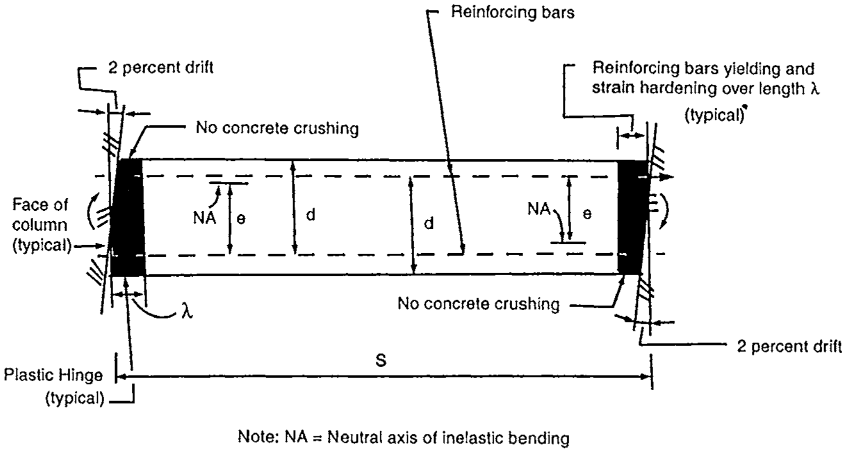
Reinforcing bar physical properties are main determinants for reinforcing-bar seismic demands. Consequently, seismic codes set appropriate single upper or lower limits on reinforcing bar yield strength and tensile/yield ratio, but they do not consider the variable-parameter effects of the shape of the reinforcing-bar stress-strain curve on what tensile/yield ratios and ductilities should realistically be required of reinforcing bars in seismic-resistant structures. Therefor, a theoretical study was performed to evaluate the effect of range of allowable steel yield strength, shape of steel stress-strain curve (strain and tangent modulus of elasticity at onset of strain hardening), and beam slenderness (S/d, where S is the clear span length and d is the effective depth to the reinforcing bar centroid, Figure 1) on the minimum values of steel tensile/yield ratio and useful ductility that are necessary to accommodate 2 % seismic drift by plastic hinging at the ends of beams, Figure 1, of concrete rigid frames reinforced with Grade 60 steel reinforcing bars.

Keywords: ductility; modulus of elasticity; reinforcing bar

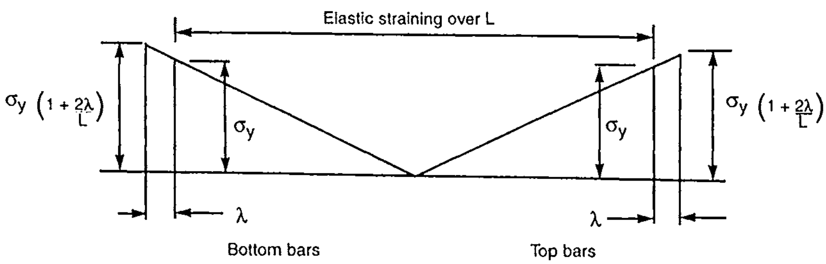
The analysis strategy included (1) relating S/d to e/L, where L is the beam elastic length and e is the depth from the reinforcing bar centroid to the neutral axis, (2) stating a reasonable definition for tensile stress along the beam, in terms of the unknown length of plastic hinging, bottom of Figure 1, (3) evaluating the half-beam elastic rotation as $(\sigma_y L)/(4Ee)$, where σ_y is the steel yield strength and E is the steel modulus of elasticity, (4) for given values of the parameters, evaluating the total plastic-hinge rotation as 2 % minus the half-beam elastic rotation, (5) expressing the strain definitions of the steel stress-strain curve in terms of a plastic proportion parameter (plastic-hinge length/beam elastic length), top of Figure 2, (6) equating the already-calculated total plastic hinge rotation to the product of the unknown plastic-hinge length times the average curvature within the plastic hinge (reinforcing bar average strain/e) to obtain a quadratic equation relating the plastic-proportion parameter, yield strength, tangent modulus of elasticity, strain ratio (s, Figure 2), e/L, and total plastic hinge rotation, bottom of Figure 2, giving a numerical evaluation of the plastic-proportion parameter, and (7) evaluating the terminal point (required-useful-ductility abscissa and required-tensile-strength ordinate) of the stress-strain curve, top of Figure 2.

The calculation results, summarized in Figure 3, indicate that the generally accepted requirement of a minimum tensile/yield ratio of 1.25 for reinforcing steel in seismic design is both prudent and probably satisfactory. However, considering the high steel strains, particularly about 0.1 in./in. for squatty beams, the study strongly suggests that it would be prudent, for seismic-resistant reinforced concrete structures, to either specify A706 steel or specify A615 steel to have at least a 10 % minimum elongation in 8 inches (203 mm). For reinforcing in very squatty beams (with close to the minimum length-to-depth ratio permitted by ACI 318-95) the calculations indicate that the stress-strain curve for the reinforcing steel should not exhibit too low a tangent modulus of elasticity at onset of strain hardening (possibly causing ductility capacity to be exceeded) or too high a tangent modulus (possibly causing tensile strength capacity to be exceeded).

Postscript. The desired useful ductility refers to a 10 % strain at only one station at the end of the plastic hinge, so that the corresponding average bar strain within the plastic hinge would be closer to 5 %. That, considering comparisons of plastic-hinge lengths (calculated in the present study to range from 10 % to 68 % times the beam effective depth) with lengths of reinforcing bar mechanical splices (e.g., ranging from about 6 to 36 inches for #18 reinforcing bars), provides reasonable justification for requiring 4 % average tensile straining to determine the loading for the cyclic testing of seismic-rated mechanical splices of reinforcing bars.

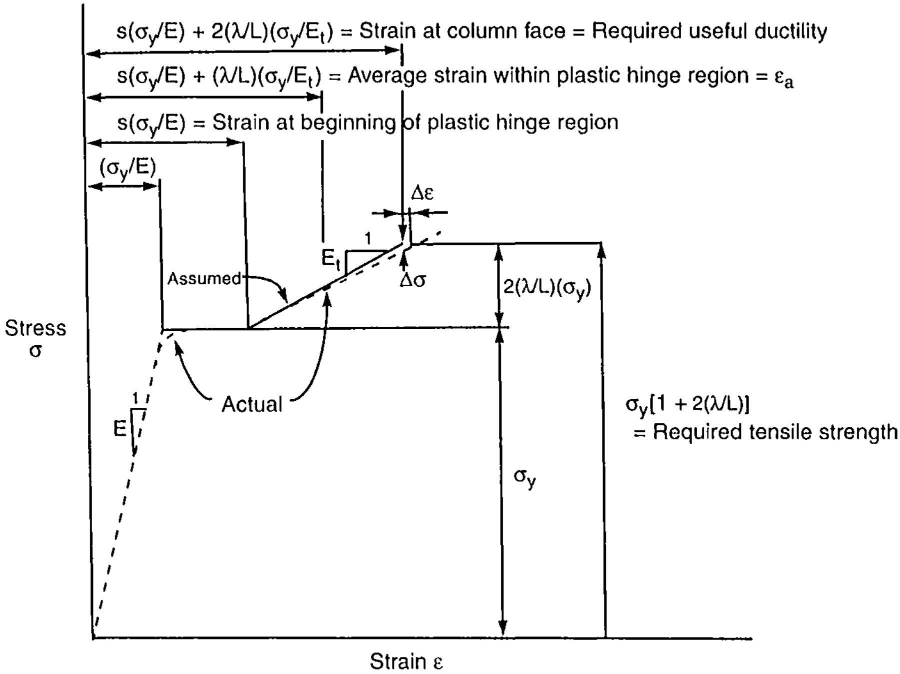


(a) Elevation view of rigid-frame beam subjected to maximum allowable column drift



(b) Assumptions for tensile stresses in reinforcing bars

Fig. 1—Simplifying assumptions for analyses.



Plastic hinge rotation, θ, assumed = $\lambda\varepsilon_a/e$, from which the portion of span length within one plastic hinge = $\lambda/L = \dfrac{\{-s(\sigma_y/E) + \sqrt{[s(\sigma_y/E)]2 + 4\theta(e/L)(\sigma_y/E_t)}\}}{2\sigma_y/E_t}$

Fig. 2—Reinforcing bar straining within region of plastic hinging.

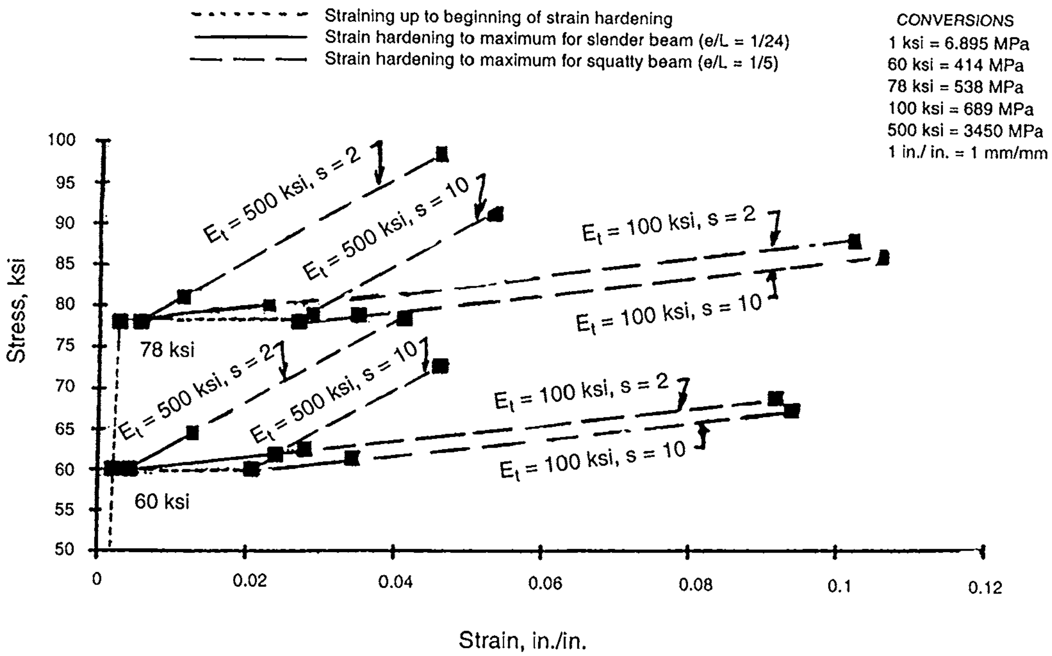


Fig. 3—Effect of beam slenderness and shape of steel stress-strain curve on seismic demands for reinforcing bar ductility and tensile strength, based on 2 percent drift.

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Design and Performance of Bridge Cap Beam/Column Joints Using Headed Reinforcement and Mechanical Couplers

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ABSTRACT

The application of headed reinforcement as a replacement for conventional reinforcement was investigated in two projects relating to the seismic design of bridges. In the first project, a test unit composed of a column, cap beam, footing and knee joint was designed entirely with headed reinforcement, and in the second project a test unit representative of a multi-column bridge bent was investigated, having a cap beam design utilizing both headed reinforcement and a mechanical coupler system. In both investigations the use of recently-developed reinforcement products facilitated simplified detailing, particularly in the cap beam/column joint region, resulting in reduced reinforcement congestion in the joint zone and improved constructability. The design and performance of the test units under simulated seismic loading are presented.

Keywords: cap beam; column; footing; reinforcement

INTRODUCTION

The 1989 Loma Prieta earthquake caused significant damage to bridge stock in the San Francisco Bay area [4]. This damage, combined with post-earthquake analysis, identified several design shortcomings in existing bridge structures [8], emphasizing the need for a critical review of California bridge seismic design procedures. Consequently, comprehensive research programs were initiated at several institutions in California investigating possible retrofit techniques for existing structural deficiencies and establishing seismic design guidelines for modern bridges.

One of the design deficiencies identified in existing bridges was inadequate detailing of cap beam/column connections, whose performance is critical at the survival limit state. Collapse of, or damage to a number of bridges in the Loma Prieta earthquake, including the double-deck Cypress viaduct, was attributed to poor detailing of the beam/column joints [4]. As outlined in the following section, when joints are detailed in accordance with the conventional design philosophy based directly on shear forces, considerable reinforcement congestion is likely. In this paper, testing conducted at the University of California at San Diego (UCSD) is used to demonstrate that simplified reinforcement details can be obtained for structural members when utilizing new reinforcement products such as headed rebars and mechanical couplers in conjunction with joint force transfer models. This significantly reduces congestion problems, particularly in cap beam/column connections, while providing satisfactory overall seismic performance for the structure.

SEISMIC DESIGN PROCEDURE

The capacity design philosophy, which now forms the basis for bridge design in most seismically active countries of the world, emphasizes ductile structural performance under severe seismic loading. In concrete bridges, ductile response is typically developed by forming plastic hinges at the top and/or bottom of bridge columns. The reinforcement located in these hinge regions is carefully detailed to accommodate large inelastic reinforcement strains and local member rotations, allowing seismic energy to be dissipated in the form of hysteretic damping. The remaining elements of the structure are protected from significant inelastic action by providing a strength hierarchy sufficient to cope with potential strain hardening and uncertainties in material strengths.

The elastic design of non-critical structural members (typically the bridge cap beam) is generally well established. However, the design of joints which the bridge members frame into is in comparison poorly understood and is not specifically addressed in bridge design codes such as AASHTO [1] and Caltrans specifications [3]. Two alternative methods may be considered for detailing bridge joints to ensure satisfactory performance complying with capacity design criteria.

1. Building Code Approach

In building codes that require specific design of joints, such as NZS 3101:1995 [2] and ACI 318-95 [12], the design of beam/column connections is based upon the maximum joint shear force which is expected at the ultimate limit state. If a similar approach is considered for the design of bridge joints, robust joint performance is ensured. However, this design procedure, when applied to bridge joints, has been found to require an unnecessarily conservative amount of reinforcement, resulting in major congestion within the joint [6,9,11].

2. Rational Force-Transfer Method

In research studies at UCSD the design of bridge joints has been investigated [5 – 11] using force transfer mechanisms which ensure a satisfactory pathway for forces through the joint. It has been shown experimentally that good seismic bridge joint response can be obtained using significantly less than the code-recommended quantity of joint reinforcement when the design is based on force transfer mechanisms.

In a well-designed bridge joint, the flexural capacity of the column, which frames into the joint, dictates the shear demand within the joint. Consequently, when a concrete bridge bent is designed with high longitudinal reinforcement content in the columns ($\rho_l \geq 2.5\%$), the required reinforcement in the joint region based on force transfer models can also create congestion problems. In such circumstances, joint reinforcement congestion can be alleviated using alternative reinforcement products as demonstrated in the two large-scale tests presented in this paper.

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[1] Headed Reinforcement Corp., 11200 Condor Ave., Fountain Valley, CA 92708.

RECENTLY-DEVELOPED REINFORCEMENT PRODUCTS

In recent years a large number of products have become available in the United States to simplify the anchorage and lap splicing of conventional reinforcement. Two such products, namely headed reinforcement and mechanical couplers (or bar extenders) were used in large-scale experiments on bridge structural systems at UCSD. These reinforcement products were designed and manufactured by Headed Reinforcement Corporation[1].

When new reinforcement products are used in seismic design, it is *not* always sufficient to assess these products based upon monotonic stress-strain response. Depending upon the design, it may be necessary to ensure that the product can withstand cyclic inelastic strains to produce a satisfactory structural response when subjected to earthquake loading. In all cases, it is required that the ultimate capacity of the reinforcement product be not less than the ultimate capacity of the parent reinforcing bar. Description and relevant laboratory tests performed on the headed reinforcement and bar extender products are as follows.

1. Headed Reinforcement

Headed reinforcement provided in the knee joint unit was manufactured by friction welding forged circular heads to conventional ASTM A706/A706M-90 grade 60 (414 MPa) weldable reinforcement (see Fig. 1a). In the process of quality-assurance tests performed by the manufacturer, the headed reinforcement exhibited cyclic behavior identical to that of the parent reinforcing bar with ultimate failure consistently occurring in the reinforcement, not at the friction weld. To verify that the full capacity would be developed in the parent reinforcing bar, a total of eight headed bars were randomly selected during construction of the knee joint unit at the UCSD facilities, and tested in uniaxial tension. In all cases, fracture of these randomly selected samples occurred in the reinforcing bar, away from the friction weld.

2. Mechanical Coupler (or Bar Extender) System

The mechanical coupler system used in the second test unit incorporated two fixtures which coupled two headed reinforcing bars using standard threads (see Fig. 1b and 1c). The reinforcement heads were formed using a technique